

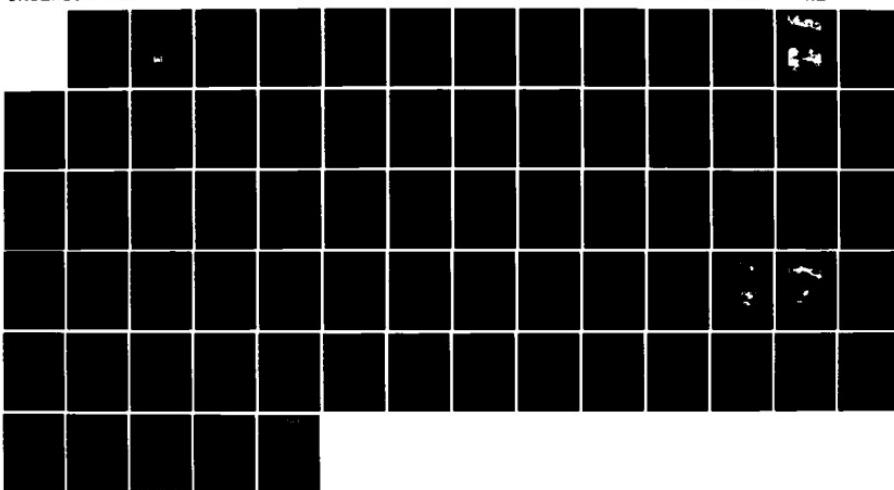
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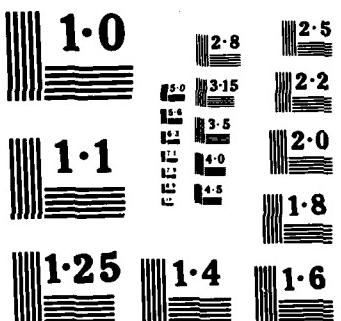
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CHESTERFIELD, NEW HAMPSHIRE

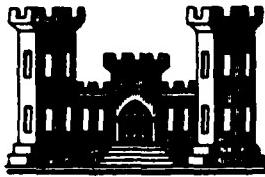
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SPOFFORD LAKE DAM

NH 00356

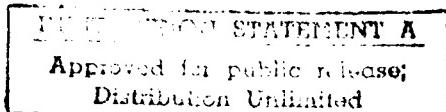
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PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM



DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

NOVEMBER 1978



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REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER NH 00356	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) Spoffard Lake Dam		5. TYPE OF REPORT & PERIOD COVERED INSPECTION REPORT
NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS		6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS DEPT. OF THE ARMY, CORPS OF ENGINEERS NEW ENGLAND DIVISION, NEDED 424 TRAPELO ROAD, WALTHAM, MA. 02254		12. REPORT DATE November 1978
14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office)		13. NUMBER OF PAGES 70
		15. SECURITY CLASS. (of this report) UNCLASSIFIED
		16a. DECLASSIFICATION/DOWNGRADING SCHEDULE
16. DISTRIBUTION STATEMENT (of this Report) APPROVAL FOR PUBLIC RELEASE: DISTRIBUTION UNLIMITED		
17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)		
18. SUPPLEMENTARY NOTES Cover program reads: Phase I Inspection Report, National Dam Inspection Program; however, the official title of the program is: National Program for Inspection of Non-Federal Dams; use cover date for date of report.		
19. KEY WORDS (Continue on reverse side if necessary and identify by block number) DAMS, INSPECTION, DAM SAFETY, Connecticut River Basin Chesterfield, New Hampshire Partridge Brook		
20. ABSTRACT (Continue on reverse side if necessary and identify by block number) The dam is an 83 ft. long earth filled masonry crib with a maximum height of about 11 ft. It is intermediate in size with a significant hazard potential. The test flood is between the $\frac{1}{2}$ PMF and full PMF. The dam is in fair condition at the present time and requires some routine maintenance. Based on the dam's fair condition, periodic technical inspections should be scheduled every two years.		

SPOFFORD LAKE DAM
NH 00356

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CHESTERFIELD, NEW HAMPSHIRE



PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

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NATIONAL DAM INSPECTION PROGRAM
PHASE I INSPECTION REPORT

Identification No.: NH 00356
NHW RB No.: 45.08
Name of Dam: SPOFFORD LAKE DAM
Town: Chesterfield
County and State: Cheshire, New Hampshire
Stream: Partridge Brook
Date of Inspection: September 21, 1978

BRIEF ASSESSMENT

The Spofford Lake Dam is an 83 foot long earth-filled masonry crib with a maximum height of approximately 11 feet. The dam's present discharge capacity consists of a 4 foot wide by 3 foot 9 inch high concrete culvert regulated by a stoplog weir. Prior to construction of the stoplog weir at the recommendation of the New Hampshire Water Resources Board (NHW RB) in 1955, the same culvert passed discharge from a still present, but abandoned, spillway and sluice gate. The precise nature of the inlet works is not important in the dam's present configuration as the actual discharge is limited by the capacity of the culvert. Historical records indicate that the dam, which is now owned by the Town of Chesterfield, was built in 1919.

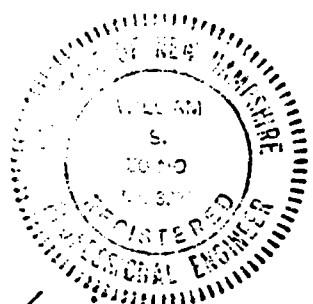
The dam lies on Partridge Brook, which drains directly into the Connecticut River, and receives runoff from 4 square miles of generally steeply sloping, forested terrain. The dam's maximum practical impoundment of approximately 5200 acre-feet places it in the INTERMEDIATE size category, while the possibility of considerable property damage, but unlikely loss of life, in the event of a failure results in a SIGNIFICANT hazard potential classification.

Based on the size and hazard potential ratings and in accordance with the Corps' guidelines, the Test Flood (TF) is between one half the Probable Maximum Flood (PMF) and the full PMF. The selected TF inflow in this case is 5500 cfs. Due the very large lake surface area as compared to the drainage area and to the large amount of freeboard provided by the earth fill, the reservoir can store the entire TF and still have over one foot of freeboard.

This situation is fortuitous, as the dam's limited discharge capacity of 145 cfs at maximum pool elevation would otherwise be a serious deficiency. It is still recommended, however, that the discharge capacity be improved to keep water off the earth portions of the embankment and to permit better control of the lake level in the event of a severe storm.

The dam is in FAIR condition at the present time and requires some routine maintenance. Included in these O & M procedures are repair of deteriorated concrete, repair of erosion, removal and trimming of vegetation on the earth fill and in the downstream channel, removal of the inoperative sluice gate, cleaning of the concrete culvert, installation of a gage, institution of a formal record of operations and lake levels and establishment of an ongoing preventative maintenance program.

The preceding recommendations and remedial measures should be accomplished within two years of receipt of the Phase I Inspection Report. Based on the dam's FAIR condition, periodic technical inspections should be scheduled every two years.



William S. Zoing
William S. Zoing
New Hampshire Registration 3226



Nicholas A. Campagna, Jr.
Nicholas A. Campagna, Jr.
California Registration 21006

This Phase I Inspection Report on Spofford Lake Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

Charles G. Tiersch

CHARLES G. TIERSCH, Chairman
Chief, Foundation and Materials Branch
Engineering Division

Fred J. Ravens Jr.

FRED J. RAVENS, Jr., Member
Chief, Design Branch
Engineering Division

Saul Cooper

SAUL COOPER, Member
Chief, Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

Joe B. Tryar

JOE B. TRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the Test Flood should not be interpreted as necessarily posing a highly inadequate condition. The Test Flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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Overview of dam from left abutment showing
gate house and approach channel



Overview of dam from right side upstream
showing inlet area



- SCALE -

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FROM: USGS KEENE, N.H.
QUADRANGLE MAP

GOLDBERG, ZOINO, DUNNCLIFF & ASSOC, INC.
GEOTECHNICAL CONSULTANTS
NEWTON UPPER FALLS, MASS.

U.S. ARMY ENGINEER DIV NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS

LOCUS PLAN

FILE NO. 2067

SPOFFORD LAKE DAM

NEW HAMPSHIRE

SCALE AS NOTED
DATE SEPT 1978

PHASE I INSPECTION REPORT

SPOFFORD LAKE DAM

SECTION 1

PROJECT INFORMATION

1.1 General

(a) Authority

Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a national program of dam inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Goldberg, Zoino, Dunnicliff & Associates, Inc. (GZD) has been retained by the New England Division to inspect and report on selected dams in the State of New Hampshire. Authorization and notice to proceed was issued to GZD under a letter of August 22, 1978 from Colonel Ralph T. Garver, Corps of Engineers. Contract No. DACW 33-78-C-0303 has been assigned by the Corps of Engineers for this work.

(b) Purpose

- (1) Perform technical inspection and evaluation of non-federal dams to identify conditions which threaten the public safety and thus permit correction in a timely manner by non-federal interests.
- (2) Encourage and prepare the states to initiate quickly effective dam safety programs for non-federal dams.
- (3) Update, verify and complete the National Inventory of Dams.

(c) Scope

The program provides for the inspection of non-federal dams in the high hazard potential category based upon location of the dams and those dams in the significant hazard potential category believed to represent an immediate danger based on condition of the dam.

1.2 Description of Project

(a) Location

Spofford Lake Dam lies on Partridge Brook in the village of Spofford, NH. Canal Street runs directly over the dam and the site is easily reached via Routes 9 and 9A. The portion of the USGS Keene, NH quadrangle presented on page viii shows this locus. Figure 1 of Appendix B presents a site plan developed from historical records and the site visit.

(b) Description of Dam and Appurtenances

The dam is basically an 83 foot long earth-filled masonry crib with a concrete wall serving as the upstream face and a squared stone masonry wall forming the downstream face (Figures 2, 4 and 5). A considerable quantity of loose gravel has been dumped in front of the upstream wall (Figure 5). The dam is approximately 28 feet wide and has a maximum height of almost 11 feet (Figure 4). Discharge at the dam is through a concrete culvert which is 4 feet wide by 3 feet 9 inches high at the upstream concrete wall and which decrease slightly in size as it passes under Canal Street. A steel sluice gate, still in place, but badly deteriorated, once controlled flow into the culvert. A 4 foot 9 inch wide stoplog weir, installed on the recommendation of the New Hampshire Water Resources Board in 1955, now performs this function. The dam has no functional spillway in its present configuration.

(c) Size Classification

The dam's maximum impoundment of over 6000 acre-feet falls within the 1000 to 50,000 acre-feet range which defines INTERMEDIATE size category as defined in the "Recommended Guidelines." While such a storage volume could, based on the topography of the area and the height of the dam, be retained, the intended maximum capacity is probably closer to 5200 acre-feet.

(d) Hazard Potential Classification

While the dam has a generally steep and well defined downstream channel, several old buildings are located immediately upon the stream banks.

Additionally, as the channel winds through Spofford, it flows under several locally important roads which could be washed out in a major flood. However, while property damage is likely to be considerable, no loss of life is anticipated. For these reasons, a SIGNIFICANT hazard potential classification is assigned.

(e) Owner

The Town of Chesterfield owns the dam. Spofford Fire Chief Ronald Guyette is the local official responsible for the dam. He lives in Spofford and can be reached at (603) 363-4411.

(f) Operator

Chief Guyette is also the dam operator. Mr. Frank Woodbury, Box 65, Spofford, who lives immediately adjacent to the dam, can also operate the dam in the event of an emergency.

(g) Purpose of Dam

The dam's primary purpose is to maintain the level of Spofford Lake for recreational use. Additionally, some flood control benefits are also derived.

(h) Design and Construction History

Historical records indicate that the dam was built in 1919, although it appears that some type of structure may have existed at the site somewhat earlier. In 1955, the stoplog weir was added at the suggestion of the NHWRB, which also recommended that the existing gate be removed.

(i) Normal Operational Procedure

The dam's operational capability consists of the installation and removal of up to two feet of stoplogs. During the summer months, one or two stoplogs (16 inches total) are usually in place to maintain the recreational pool. As winter approaches, the lake is slowly drawn down to a level which will minimize ice damage to shoreline properties when the lake freezes.

1.3 Pertinent Data

(a) Drainage Area

Spofford Lake receives runoff from 4 square miles of generally steeply sloping, heavy forest. The ratio of the drainage area to the lake area is very small, being only approximately 3.5:1. There is considerable development around the shore of the pond.

(b) Discharge at Damsite

(1) Outlet Works

The dam's only outlet is the concrete culvert under Canal Street. The culvert opening measures 4 feet wide by 3 feet 9 inches high at the gatehouse and decreases slightly in size to approximately 4 feet by 3 feet at the outlet point. Until 1955, discharges through the culvert, which has its invert at El. 710.2, were controlled by a steel sluice gate, which is still in place, but which is badly deteriorated. In that year, a stoplog weir was constructed upstream of the gate and rendered that control feature useless (Fig. 3). The crest of the permanent portion of the weir is at the elevation of the old spillway, which is at El. 716 based on the 1958 Keene, NH quadrangle. A 6-inch diameter steel pipe with invert at El. 713.6 penetrates the permanent section of the weir and provides a continuous downstream flow. With the present configuration, the only drawdown capability below El. 716 + is the uncontrolled 6 inch diameter pipe, which could not significantly influence the level of the lake given normal runoff into the impoundment.

(2) Maximum Known Flood at damsite

No official data on experienced peak flood flows or lake levels are available for this dam. Local residents indicate, however, that the lake level has not overtopped the upstream concrete wall in the last 20 years.

(3) Stoplog weir capacity at normal pool elevation:

0 cfs at El. 716

(4) Stoplog weir capacity at maximum pool elevation:

215 cfs at El. 721.1

(5) Total discharge capacity at maximum pool elevation:

215 cfs at El. 721.1

(c) Elevation (feet above MSL based on setting permanent crest of stoplog weir at 1958 USGS lake level of 716)

- (1) Top of dam: 721.1 ±
- (2) Maximum pool: 721.1 ±
- (3) Recreational pool: 716 ±
- (4) Spillway crest: 716 ±
- (5) Streambed at centerline of dam: 710.2 +
- (6) Maximum tailwater: Unknown

(d) Reservoir

- (1) Length of recreational pool: 1.7 miles ±
- (2) Storage - recreational pool: 2800 acre-feet
- maximum pool: 6460 acre-feet ±
- (3) Surface area - recreational pool: 718 acres ±

(e) Dam

- (1) Type: Earth-filled masonry crib with stoplog weir discharging into concrete culvert
- (2) Length: 83 feet
- (3) Height: 10.9 feet ±
- (4) Top Width: 28 feet ±
- (5) Side slopes - U/S 4:1 (including dumped gravel)
- D/S Vertical
- (6) Zoning, impervious core, cutoff and grout curtain: Unknown

(f) Regulating Outlet

The dam's only regulating outlet is the concrete culvert controlled by the stoplog weir. Pertinent data on this feature are contained in subparagraph 1.3(b)(1).

SECTION 2 - ENGINEERING DATA

2.1 Design

The design of this dam is somewhat unusual in that it provides only minimal discharge capacity. While the original structure did have a spillway and a gated sluiceway, both discharged into the 4 foot wide by 3 foot 9 inch high concrete culvert under Canal Street. This culvert would significantly restrict flow in the event of the Test Flood. The stoplog weir which replaced the gate as the primary control feature in 1955 did not alter this situation. No original design drawings or calculations are available, especially regarding the construction of the earth fill and foundation conditions at the site.

2.2 Construction Data

The only available information regarding the construction of the dam is a plan of the 1955 modifications recommended by the NHWRB. The drawing is of little benefit, however, as it contains significant dimensional errors and is incomplete.

2.3 Operational Records

The owner maintains no operational records for the dam. In general, whatever limited operations are possible appear to be carried out in a manner consistent with the structure's intended purpose and engineering features.

2.4 Evaluation of Data

(a) Availability

The absence of design drawings and calculations is a significant shortcoming. An overall unsatisfactory assessment for availability is, therefore, warranted.

(b) Adequacy

The lack of in-depth engineering data does not permit a definitive review. Therefore, the adequacy of the dam cannot be assessed from the standpoint of reviewing design and construction data. This assessment is thus based primarily on the visual inspection, past performance and sound engineering judgement.

(c) Validity

Since the observations of the inspection team generally confirm the information contained in the sparse written data, with modification, a satisfactory evaluation for validity is indicated.

SECTION 3 - VISUAL OBSERVATION

3.1 Findings

(a) General

The Spofford Lake Dam is in FAIR condition at the present time. Some routine maintenance is required to protect the long-term use and safety of the structure.

(b) Dam

(1) Embankment (Photo 1)

Observation of the earth-filled crib, which also forms Canal Street, revealed no evidence of horizontal or vertical movement.

The upstream face of the dam is an 18 inch thick, vertical concrete wall, but gravel has since been dumped in front of the wall on the left side of the dam to provide fire trucks access to the lake for drawing water. Only 12 to 18 inches of concrete remains exposed on the left side, compared to 2 to 3 feet on the right. The top part of the left side of the wall is significantly deteriorated, partly by erosion and partly by trucks driving over it. A few voids up to 12 square inches in area and 2 inches deep were noted on the face. Evidence of erosion and spalling is present to a minor degree at joints, while aggregate has been exposed over 50% of the surface. Random hairline cracking is evident over most of the wall. The right side of the wall is in somewhat better condition, displaying only minor erosion and random hairline cracking. There is no evidence of cracking or sloughing of the fill near the upstream wall.

The downstream face of the dam is an open jointed, squared stone masonry structure. The wall is approximately 1.5 to 2 feet thick at the top. Neither displacement nor seepage through the wall was observed. There is no evidence of distress in the fill behind or at the toe of the wall.

There is light brush growing on both shoulders of the road.

Heavier brush and several small trees are growing in front of the right side of the upstream wall and on the upstream side of the left abutment.

No piping or boiling was noted. The only available plan shows no foundation drainage features and none are evident.

There is no rock protection anywhere on the earth fill and no evidence of any erosion of soil. No problems were noted at the abutment areas. The asphalt concrete road pavement over the dam is in fair condition, displaying considerable alligator cracking, particularly at the edges.

No information as to the internal construction of the earth fill or to the foundation conditions at the site is available.

(2) Outlet Works (Photos 2 and 3)

The dam lies on a broad approach channel from the lake. There appears to be no nearby trees or other overhanging vegetation which might block the dam if felled in a storm. There is a small amount of floating debris against the trashrack. Additionally, this inlet has, in the past, been blocked by beaver placed debris requiring occasional cleaning by the owner.

The inlet works consist of two 18 inch thick concrete walls with a trashrack on the upstream end. The stoplog weir is approximately 2 feet behind the trashrack. The walls extend past the weir and joint the concrete wall which forms the upstream face of the dam. Two 5 foot long wing walls extend parallel to the upstream concrete wall in the plane of the stoplog weir. A metal grate spans between the two inlet walls and over the weir.

The trashrack is in good condition and displays only minor surface rusting. The horizontal grate is in similar condition. Both are locked in place, preventing unauthorized access to the stoplogs.

The two inlet walls are in fair condition despite surface erosion exposing aggregate, minor spalling, random hairline cracking and some deterioration at joints. The soil under the walls has been eroded to a depth of at least 6 inches.

The two smaller walls parallel to the front face of the dam display identical deficiencies. Erosion is occurring at the ends of these walls, resulting in 6 to 12 inches of the walls acting as a cantilever.

The concrete forming the permanent section of the stoplog weir and the stoplogs themselves are in good condition. Minor surface erosion is evident on the concrete. The uncontrolled 6 inch diameter pipe through the concrete is in good condition.

Prior to the construction of the stoplog weir in 1955, the outlet works consisted of an ogee type spillway and a steel sluice gate, both discharging into the culvert under the road. The steel gate is now deteriorated and cannot be moved from the open position. Due to the presence of the weir and the open condition of the gate, the old spillway is also functionally useless and suffers from severe deterioration of its concrete cover. Apparently capped sometime in the past, erosion now penetrates up to 2 inches into the ogee section.

All discharge from the dam must pass under the road through the 4 foot by 3 foot 9 inch concrete culvert. Construction details of the culvert are not evident and are not shown on the old plan. The side and top of the structure are in fair condition, displaying only minor cracking, spalling and surface erosion. Some erosion is taking place at the junction of the sidewalls and the bottom slab and the bottom of the culvert is covered with up to 6 inches of sand and gravel. The downstream end of the culvert is severely restricted by boulders and siltation, having only approximately 1 foot of vertical clearance. The immediate discharge channel is similarly obstructed with boulders and debris extending above the top of the opening.

(c) Appurtenant Structures (Overview Photos)

The wooden gate house is in good condition with the exception of the timber floor, which is deteriorated and unsafe. There is no gage at the dam.

(d) Downstream Channel (Photo 4)

The channel downstream of the dam follows 2 courses, one of which may have once carried water to a mill. The channel is fairly steep and generally well defined, but has heavy vegetation growing in and all around it. A few buildings are constructed directly on the banks and could be affected in the event of dam failure. The channel passes under several local roads, including Route 9A twice. There are no obstructions in the channel which would adversely affect the operation of the dam, particularly in light of the dam's limited discharge capacity.

(e) Reservoir

Examination of the reservoir revealed no instability or potential for slides. No sedimentation was noted behind the dam. Additionally, no work in progress or recently completed which might increase the flow of sediment into the lake was evident. There were no apparent changes to the surrounding area which might adversely affect the runoff characteristics of the basin. There is a considerable amount of development, both year round and seasonal, around the lake.

3.2 Evaluation

The Spofford Lake Dam is in FAIR condition at the present time and requires only routine maintenance to protect its long-term use and safety. As most of the dam's key components were sufficiently accessible for examination, the visual inspection provided a satisfactory basis for evaluation.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures

The dam's operational capability is limited to the placement or removal of up to 2 feet of stoplogs. In general, the stoplogs are installed in the summer to maintain the lake level for recreational purposes and removed in the winter to prevent ice damage around the shore.

4.2 Maintenance of Dam

No formal maintenance program is established for the dam. It also appears that no maintenance work has been accomplished recently.

4.3 Maintenance of Operating Facilities

The deteriorated condition of the no longer used sluice gate was probably due to lack of maintenance. The stoplog weir which replaced it requires essentially no attention other than periodic inspection of the logs and cleaning of the trash-racks.

4.4 Description of Any Warning System in Effect

No formal warning system exists for the dam.

4.5 Evaluation

The lack of routine maintenance on the dam is obvious and requires correction. Based on the hydraulic and hydrologic characteristics of the lake, as discussed in Section 5, the lack of a formal warning system is not considered a significant problem.

SECTION 5 - HYDROLOGIC/HYDRAULIC

5.1 Evaluation of Features

(a) Available Data

Data sources available for Spofford Lake Dam include several prior inventory and inspection reports. The New Hampshire Water Resources Board's "Site Evaluation Data," dated August 1975, and the New Hampshire Water Control Commission's "Data on Dams in New Hampshire," dated 1938 contain much of the basic data for the site. Available information also includes a drawing of Spofford Lake dam repairs dated September 27, 1955 and several letters regarding the operating policy of the dam.

(b) Experience Data

No data on experienced peak flood flows or lake levels other than that mentioned in subparagraph 1.3 are known to be available for Spofford Lake.

(c) Visual Observation

The dam at Spofford Lake is formed by a roadway embankment that creates a dike at the location of the lake's natural outlet. This embankment extends for a length of about 83 feet between the banks of the stream with a maximum height of about 11 feet above the stream bottom. The upstream face of the embankment has a concrete retaining wall that may have been part of an old dam that predates the roadway embankment. A concrete inlet channel to a gate house extends out from this wall. The downstream face of the embankment is a vertical, squared-stone masonry wall.

The outlet works consist of a 48 inch wide by 45 inch high concrete culvert through the roadway embankment with several interesting control mechanisms at the entrance to the culvert. Flow out of the lake was originally controlled by a hand operated sluice gate at a 30 inch by 24 inch opening into the culvert. Immediately above this opening, a 50 inch wide emergency spillway would allow excess flow to by-pass the gate. The gate is now in a deteriorated and inoperable condition. Modifications to the outlet in 1955 (from plans of that date) involved the construction of a 4.75 foot wide weir just ahead of the old gate works.

This weir has a permanent concrete base whose height has been used as an elevation reference for the entire structure by assuming that the elevation of the permanent crest corresponds to the normal pond elevation of 716.0 feet shown on the USGS topographic map. Additional height for this weir may be attained through the addition of stoplogs above the concrete portion. A 6 inch diameter pipe through the concrete base with an invert elevation of 713.6 feet provides a permanent opening and continuous flow through the outlet. At the time of the field inspection of this structure on September 20, 1978 the pond level was just overtopping a single stoplog in the weir at an elevation of about 716.5 feet.

Another important hydraulic aspect of the dam's outlet works is the condition of the concrete culvert at the downstream side of the embankment. The outlet portion of the culvert was observed to be seriously blocked by boulders and sediment with an outlet height of only about 1'8" above stream bottom. Presumably the obstruction extends for a considerable distance back into the culvert. This represents a severe reduction in the flow capacity of the culvert.

Downstream of the dam, the stream drops by about 20 feet in about a quarter mile before reaching the village of Spofford, N.H. It then passes through several small roadway culverts before becoming very steeply sloping beyond the town.

(d) Overtopping Potential

The hydrologic conditions of interest in this Phase I investigation are those required to assess the dams overtopping potential and its ability to safely allow an appropriately large flood to pass. This analysis requires use of the storage and discharge characteristics of the structure to evaluate the impact of an appropriately sized Test Flood.

Guidelines for establishing a recommended Test Flood based on the size and hazard potential classifications of a dam are specified in the "Recommended Guidelines" of the Corps of Engineers (COE). As specified in these "Guidelines," the appropriate Test Flood for a dam classified as INTERMEDIATE in size with a SIGNIFICANT hazard potential would be between one-half the Probable Maximum Flood (PMF) and the full PMF.

The chart of "Maximum Probable Peak Flow Rates" obtained from the COE, New England Division is used to define the applicable PMF. For the 3.95 square mile drainage area to Spofford Lake and a hilly or "rolling" topography, the chart gives a PMF flow of 1925 cfs per square mile. This results in a total PMF flows of 7600 cfs and a one-half PMF flow of 3800 cfs.

The "Recommended Guidelines" suggest that if a range of values is indicated for the Test Flood, the magnitude most closely relating to the involved risk should be selected. Therefore, a value of 5500 cfs is selected as inflow to Spofford Lake, which is considered to be in the middle of the SIGNIFICANT category.

In evaluating the overtopping potential of the dam it was determined that due to the drainage area and storage characteristics of Spofford Lake, the entire Test Flood volume can be stored without overtopping the dam. The maximum water level in the lake that would occur is therefore of primary interest. This level depends on several factors including volume of runoff, storage available, and outflow capacity.

The flow characteristics of the outlet are complex since different parts of the structure control outflow at different lake levels. At the higher lake levels of interest here, it is assumed that the upstream section of the outflow structure is totally submerged and that the rate of flow is controlled by the constriction at the downstream end of the culvert. Hydraulic calculations to account for the volume of inflow, the lake storage capacity, and the outflow rate as determined by the headwater and tailwater levels indicates that the maximum stage that would develop in the lake as a result of the Test Flood flow would be to an elevation of 720.0 feet. This is 0.6 feet above the concrete retaining wall, but 1.1 feet below the roadway crest. The computed lake outflow at this level is about 195 cfs.

5.2 Hydrologic/Hydraulic Evaluation

The results of the hydrologic and hydraulic calculations indicate that although the outlet capacity of Spofford Lake Dam is small, there is sufficient lake storage and freeboard to the top of the dam to prevent overtopping for the selected Test Flood.

For the Test Flood inflow to the lake of 5500 cfs which is between one-half and the full PMF flow, the maximum stage that would develop was determined to still leave about 1.1 feet of freeboard below the top of the roadway embankment which acts as the dam crest.

Hydraulically, the capacity of the outlet is severely restricted by the condition of the discharge end of the culvert and the immediate downstream channel. This capacity could be greatly improved if the boulders, debris, and sediment were removed from the end of the culvert and the downstream channel.

5.3 Downstream Dam Failure Hazard Estimates

The flood hazards in downstream areas resulting from a failure of Spofford Lake Dam are estimated based on the procedure suggested in the COE New England Division's April 1978 "Rule of Thumb Guidelines for Estimating Downstream Dam Failure Hydrographs." This procedure has been supplemented with a steady state analysis of two of the reaches to reflect their particular situations, i.e., roadway culverts at the downstream ends of these reaches that control the outflow and water levels in these reaches.

Failure of the dam is assumed to occur with a lake level at the top of the concrete retaining wall, or an elevation of 719.4 feet. This level corresponds to a height of 9.2 feet above the stream bed. For an assumed breach width of 30 feet, the resulting peak dam failure discharge is computed as 1200 cfs.

Partridge Brook (the water course downstream of the dam) is considered in three reaches downstream of the Spofford Lake Dam. The first reach is relatively steep with a fairly well defined channel and a length of 800 feet. Reach two is also 800 feet long and has a relatively flat, wide channel. The stage in this reach is controlled by the capacity of the culvert beneath the roadway at the end of the reach. The third reach has a well defined channel, is the steepest of the three, and is about 700 feet in length. It too has a roadway culvert at its downstream end which controls the stage in the reach.

To analyze those reaches the peak discharge was assumed to occur for a long enough period of time for a steady-state condition to become established, which is a valid assumption since the storage behind the dam is large enough to maintain a relatively constant lake level for a while after the dam is breached.

Thus, no attenuation of the initial peak discharge is accounted for and the flow through each reach is 1200 cfs.

The stage in reaches two and three were estimated using the Federal Highway Administration (FHWA) nomographs for flow through roadway culverts and a standard weir equation for the flow over the roadway. These relationships were used to converge on the water levels in the reaches. The results are that the roadway at the bottom of the second reach is overtopped by 1.36 feet. The roadway at the bottom of Reach 3 is overtopped by 1.90 feet.

Several low lying structures between the roadway crossings along Reach 3 would be flooded by these depths of flow. Although the depths would be sufficient to cause significant property damage, it is unlikely that the flood would endanger lives. Just below the culvert at the end of Reach 3, an old mill building now apparently used as a residence would also be within the hazard area. Because of its location, overhanging a deep ravine, it could be demolished by this flooding situation with obvious hazard to any occupants at the time. Downstream of this location the stream channel becomes quite steep with no structures endangered.

SECTION 6 - STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

(a) Visual Observations

The field investigation revealed no significant displacements or distress which warrant the preparation of structural stability calculations based on assumed sectional properties and engineering factors.

(b) Design and Construction Data

There are no design drawings or calculations available which would be of value to a stability analysis were one deemed necessary.

(c) Operating Records

There are no formal operating records for this dam. Thus, no information concerning the stability of the dam during periods of high flow is available.

(d) Post-Construction Changes

The addition of the stoplog weir in 1955, which is the only known post-construction change, would have little effect on the overall structural stability of the dam, as the concrete culvert still controls the maximum possible discharge.

(e) Seismic Stability

The dam is located in Seismic Zone No. 2 and, in accordance with recommended Phase I guidelines, does not warrant seismic analyses.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS
AND REMEDIAL MEASURES

7.1 Dam Assessment

(a) Condition

The Spofford Lake Dam is in FAIR condition at the present time.

(b) Adequacy of Information

The lack of in-depth engineering data does not permit a definitive review. Therefore, the adequacy of the dam cannot be assessed from the standpoint of reviewing design and construction data. This assessment is thus based primarily on the visual inspection, past performance and sound engineering judgement.

(c) Urgency

The remedial measures recommended below should be accomplished within two years of receipt of the Phase I Inspection Report by the owner.

(d) Need for Additional Investigation

No additional investigations are indicated at this time.

7.2 Recommendations

It is recommended that studies be made by a professional engineer to increase the discharge capacity of the dam to limit the maximum lake level in a severe storm to the level of the upstream concrete wall, or El. 719.4. While the top of the road fill is some 1.7 feet higher, it is possible that this portion of the structure could be damaged needlessly by high lake levels. Additionally, the increased discharge capacity would permit better regulation of the lake, thus decreasing complaints from littoral and downstream property owners, and would reduce flood levels upstream in the event of a severe storm.

7.3 Remedial Measures

The Spofford Lake dam requires the following operating and maintenance improvements:

- (1) Repair all eroded, spalled and cracked concrete, paying particular attention to joints.
- (2) Repair all soil erosion under concrete elements and place appropriate protection to preclude similar problems in the future.
- (3) Clear all brush and trees from the dam and the abutments.
- (4) Keep the trashrack and pipe screen free of debris to the extent possible.
- (5) Remove the inoperative and deteriorated sluice gate as recommended by the NHWRB in 1955.
- (6) Clear the downstream end of the concrete culvert and the immediate discharge area to permit maximum flow if required.
- (7) Remove excessive vegetation and debris from the downstream channel and trim or remove overhanging trees to eliminate potential obstructions to flow.
- (8) Prohibit the driving of vehicles over the upstream concrete wall as this action could damage and/or displace the wall.
- (9) Install a gage at the dam and institute a regular program of recording lake levels and stoplog operations.
- (10) Institute a regular program of preventative maintenance to preclude minor deficiencies from becoming major repairs or safety hazards.
- (11) Replace the deteriorated timber floor in the gate house.
- (12) Establish a formal warning system during periods of potential flooding.
- (13) Perform a technical inspection of the dam every two years.

7.4 Alternatives

There are no meaningful alternatives to the improvements recommended above.

APPENDIX A
VISUAL INSPECTION CHECKLIST

INSPECTION TEAM CHECKLIST

Date: September 21, 1978

NH 00356
SPOFFORD LAKE DAM
Chesterfield, New Hampshire
Partridge Brook
NHWRB 45.08

Weather: Cloudy and cool

INSPECTION TEAM

Robert Minutoli	Goldberg, Zoino, Dunnicliff & Associates, Inc. (GZD)	Team Captain
William S. Zoino	GZD	Soils
Nicholas Campagna	GZD	Soils
Andrew Christo	Andrew Christo Engineers (ACE)	Structural
Paul Razgha	ACE	Structural
Richard Laramie	Resource Analysis, Inc.	Hydrology

A son of Mr. Ronald Guyette, Spofford fire chief and the town official responsible for the dam, accompanied the inspection team.

CHECK LISTS FOR VISUAL INSPECTION		
AREA EVALUATED	BY	CONDITION & REMARKS
EARTH-FILLED CRIB		
Vertical alignment and movement	<i>NAC</i>	No deficiencies noted
Horizontal alignment and movement		No deficiencies noted: top width 28 +
Condition at abutments		No deficiencies noted
Trespassing on slopes		Vertical face downstream (D/S); loose gravel dumped on upstream (U/S) side of vertical Concrete wall to permit access to lake by fire trucks to fill up tanks
Sloughing or erosion of slopes or abutments		None noted
Rock slope protection		D/S face vertical squared stone masonry; U/S vertical concrete face covered with loose dumped gravel
Unusual movement or cracking at or near toe		None noted
Unusual downstream seepage		None noted
Piping or boils		None noted
Foundation drainage features		None evident or shown on only available plan
Upstream concrete wall		
General condition	<i>NAC</i>	Poor; only upper 12-18" exposed

CHECK LISTS FOR VISUAL INSPECTION		
AREA EVALUATED	BY	CONDITION & REMARKS
Erosion or cavitation	PJ	Upper 6-8" of left side severely eroded by water and damaged by trucks driving over edge; minor surface erosion over 5%; several horizontal voids 1-2" deep; right side fair with only minor surface erosion
Spalling		Minor spalls probably due to freeze-thaw action
Cracking		Random hairline cracking over entire wall
Condition of joints		Erosion and spalling at some joints
Rusting or staining		None noted
Visible reinforcing		None noted
Seepage or efflorescence		None noted
Downstream squared stone masonry wall		No deficiencies noted; open jointed stone
Maintenance of slopes	PJ	Light brush and small trees on right side of dam and near left abutment
OUTLET WORKS		
a. Approach Channel		
Bottom conditions	AS	Gravelly; some debris
Rock slides or falls	AS	No rock; flat shoreline
Log boom	PJ	None required

September 21, 1978
NH 00356

CHECK LISTS FOR VISUAL INSPECTION		
AREA EVALUATED	BY	CONDITION & REMARKS
Control of debris		Some debris around channel and inlet structure; restaurant overhanging right side of channel; occasionally blocked by beaver placed debris
Trees overhanging channel		None other than brush previously noted as growing on embankment
b. Old Spillway (inside gate house)	PL	
Condition of concrete		
General condition		Poor
Erosion or cavitation		Surface once capped, now eroded up to 2 inches deep
Spalling		None noted
Cracking		Minor fine random cracking
Condition of joints		No deficiencies noted
Rusting or staining		None noted
Visible reinforcing		None noted
Seepage or efflorescence		None noted
c. Stoplog weir		
Condition of concrete	PL	Fair

September 21, 1978
NH 00356

CHECK LISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REMARKS
General condition	PB	
Erosion or cavitation		Exposed aggregate on interior and exterior of inlet walls (U/S and D/S of stoplogs) and on sidewalls perpendicular to inlet walls; some erosion under inlet walls and sidewalls
Spalling		Minor spalls up to 6 square inches
Cracking		Minor hairline cracking
Condition of joints		Some erosion and spalling at joints
Rusting or staining		None noted
Visible reinforcing		None noted
Condition of stoplogs		Good
Trashrack and grate		Good condition; minor surface rust
Adequately secured (tamperproof)		Locked grate spanning over inlet walls protects weir
6" pipe		Good condition; protected by screen
d. Old gate	DV	Badly deteriorated; guides no longer serviceable; mechanism inoperative; kept in permanent open position

CHECK LISTS FOR VISUAL INSPECTION		
AREA EVALUATED	BY	CONDITION & REMARKS
e. Concrete culvert	PL	
Condition of concrete		
General condition		Fair
Erosion or cavitation		Minor surface erosion; some erosion at junction of side walls with base slab
Spalling		Minor spalls up to 6 square inches
Cracking		Minor random cracking along top of culvert and sidewalls
Condition of joints		No deficiencies noted
Rusting or staining		None
Seepage or efflorescence		Minor efflorescence around some of the random cracks in the top and in the sidewalls
Visible reinforcing		None noted
Outlet area		Discharge end of culvert 60 to 70% blocked by boulders and siltation, creating a significant flow restriction
f. Gatehouse		Good exterior condition; floor deteriorated and dangerous; serves no purpose as stoplog weir outside
g. Existence of gage	PL	None

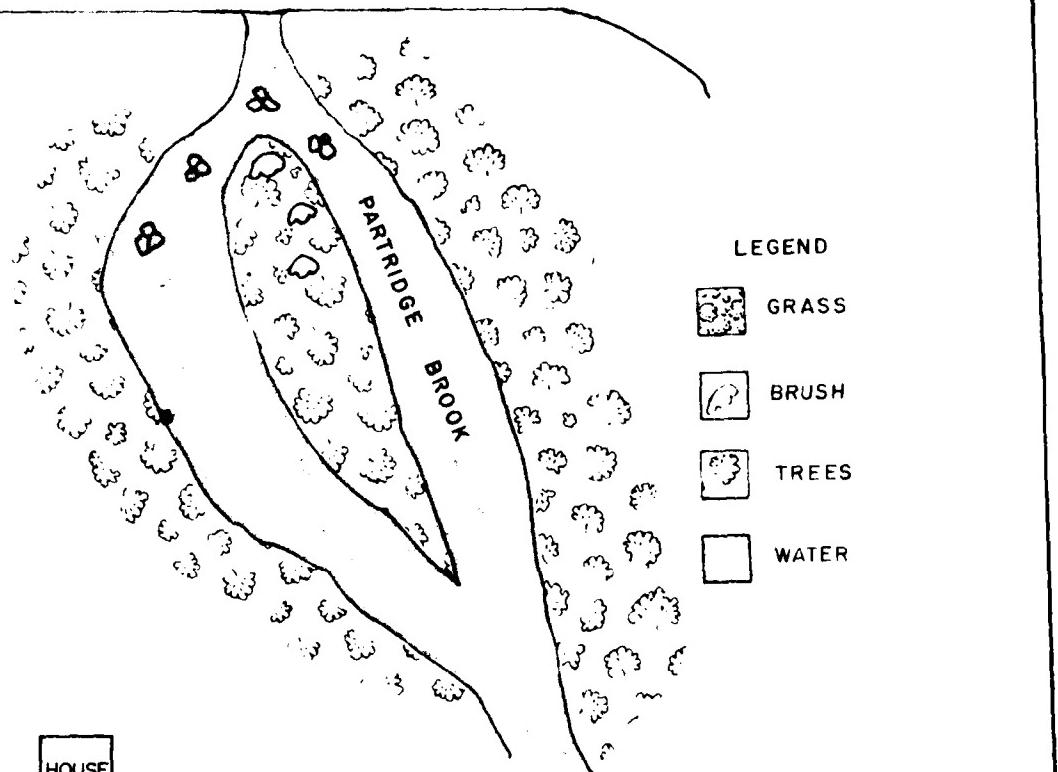
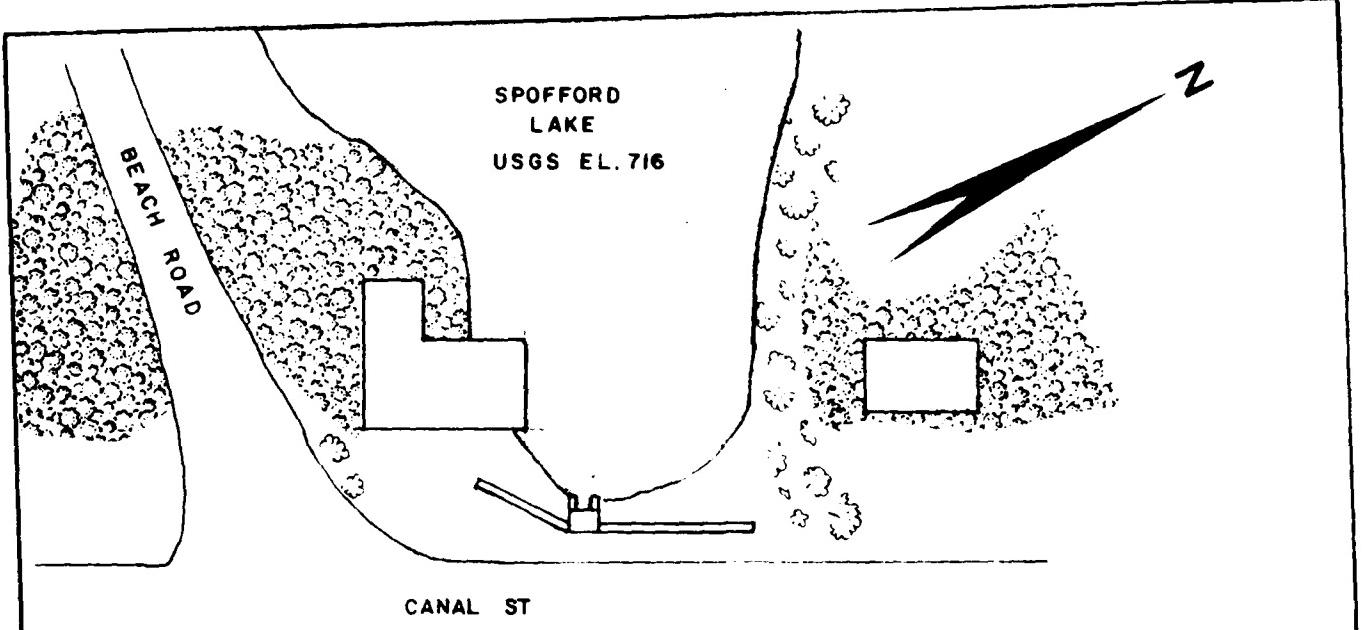
CHECK LISTS FOR VISUAL INSPECTION		
AREA EVALUATED	BY	CONDITION & REMARKS
RESERVOIR		
a. Shoreline		None noted
Evidence of slides		
Potential for slides		Shoreline stable
b. Sedimentation		Some in approach channel
c. Upstream hazard areas in the event of backflooding		Numerous permanent and summer residences immediately on shoreline of lake
d. Changes in nature of watershed (logging, construction, agriculture, etc.)		None noted
DOWNSTREAM CHANNEL		
Restraints on dam operation	MAC	None, given dam's limited operational capability; D/S channel heavily overgrown; some trees and brush could become obstructions in a storm
Potential flooded areas		Some buildings could sustain structural damage; stream crosses and could wash out several local roads and possibly Route 9A
OPERATION & MAINTENANCE FEATURES		
a. Reservoir regulation plan	MAC	

September 21, 1978
NH 00356

CHECK LISTS FOR VISUAL INSPECTION		
AREA EVALUATED	BY	CONDITION & REMARKS
Normal procedures	<i>fr</i>	Maintain lake at or above El. 716 during summer recreational period; draw down to prevent ice damage in the winter months
Emergency procedures		None; emergency operational capabilities limited to pulling stoplogs down to El. 716
Compliance with designated plan		Difficult to assess; recent (1976) complaints on file concerning ice damage and insufficient swimming depths
b. Maintenance		
Quality		Dam requires considerable routine maintenance
Adequacy	<i>fr</i>	Situation indicates a more rigorous program needed

APPENDIX B

		<u>Page</u>
FIGURE 1	Site plan	B-2
FIGURE 2	Plan of dam	B-3
FIGURE 3	Plan of inlet structure	B-4
FIGURE 4	Section through gate house	B-5
FIGURE 5	Section through earth-filled masonry crib	B-6
FIGURE 6	Details	B-7
	List of pertinent documents not included and their location	B-8



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NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS

FIG. 1

SITE PLAN

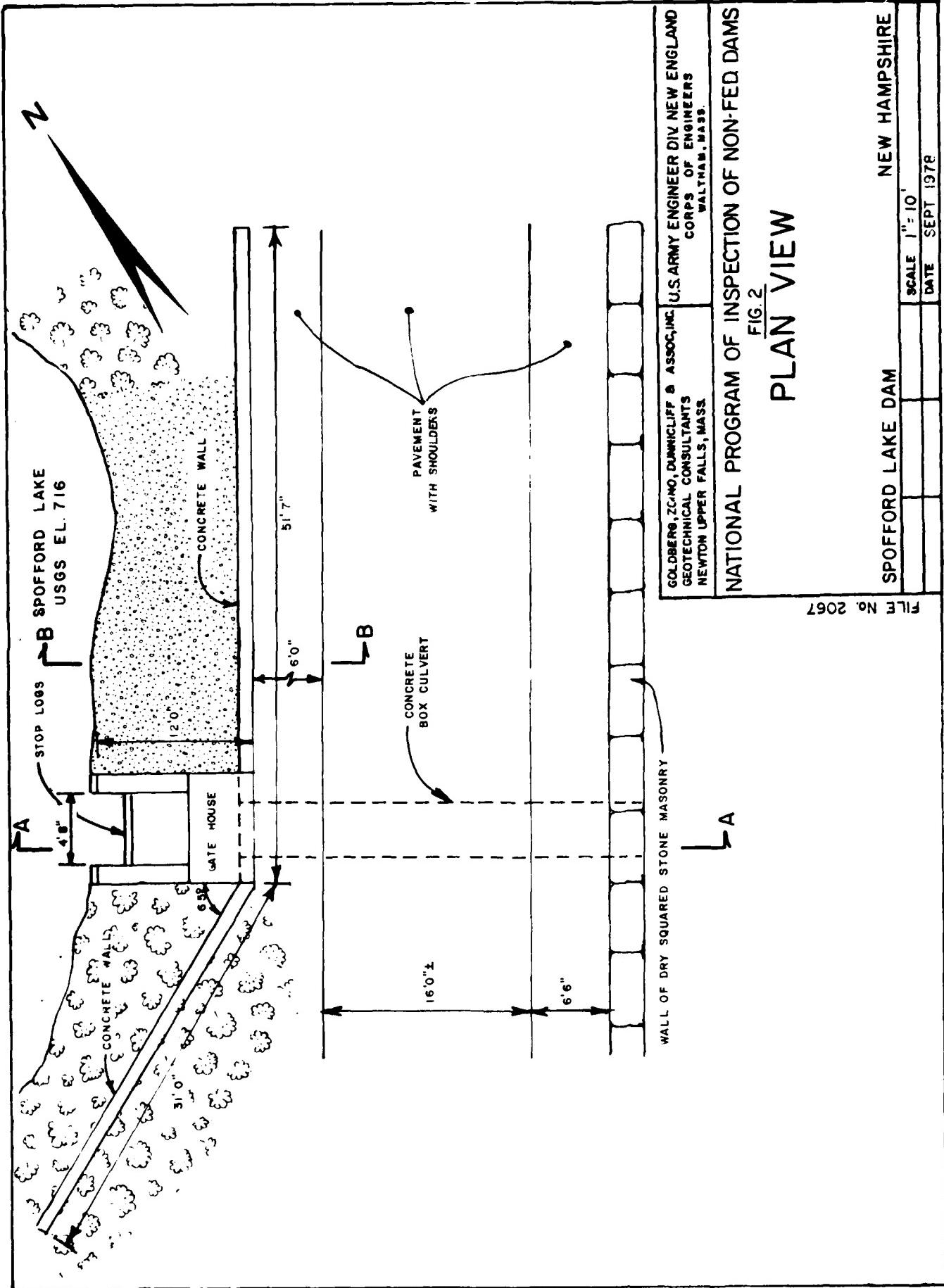
FILE NO. 2067

SPOFFORD LAKE DAM

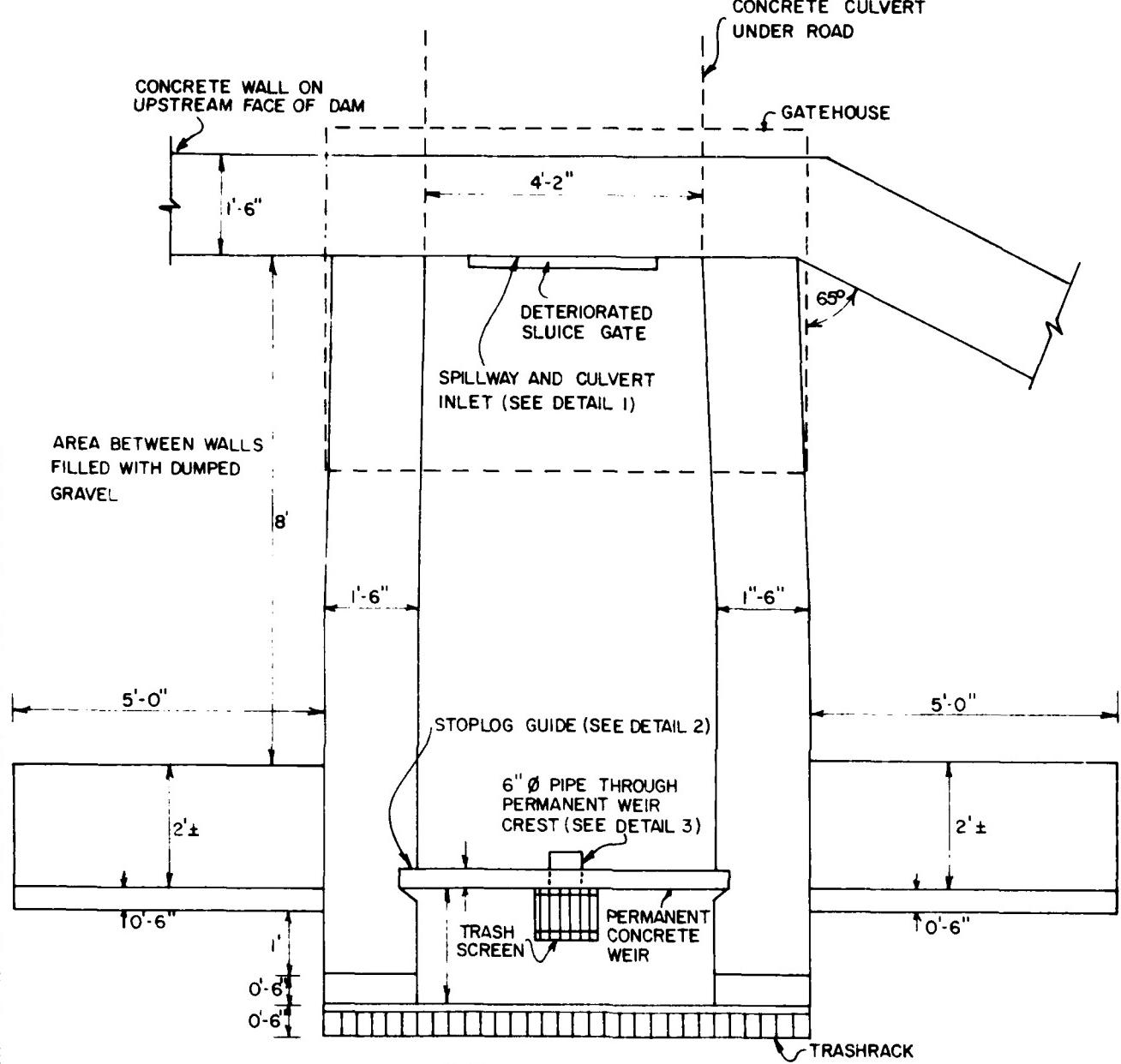
NEW HAMPSHIRE

SCALE	1" = 50'
DATE	SEPT 1978

B-2



B-3



GOLDBERG, ZINO, DUNNCLIFF & ASSOC, INC. GEOTECHNICAL CONSULTANTS NEWTON UPPER FALLS, MASS.	U.S. ARMY ENGINEER DIV NEW ENGLAND CORPS OF ENGINEERS
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NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

FIG. 3

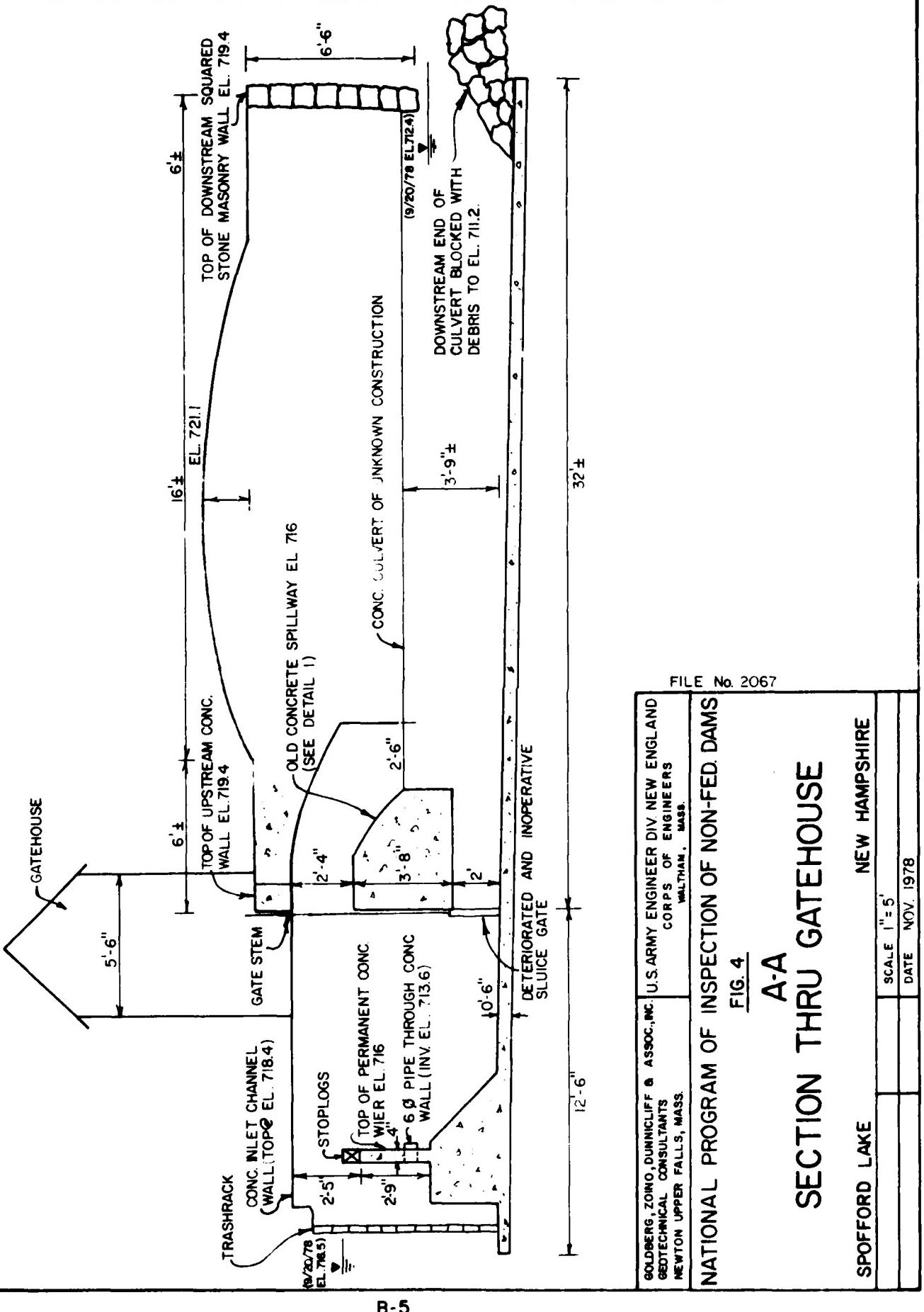
PLAN OF INLET STRUCTURE

FILE No. 2067

SPOFFORD LAKE

NEW HAMPSHIRE

SCALE	1" = 2'-6"
DATE	NOV. 1978



FILE No. 2067

GOLDBERG, ZONNO, DUNNCLIFF & ASSOC., INC. U.S. ARMY ENGINEER DIV NEW ENGLAND
GEOTECHNICAL CONSULTANTS CORPS OF ENGINEERS
NEWTON UPPER FALLS, MASS. WALTHAM, MASS.

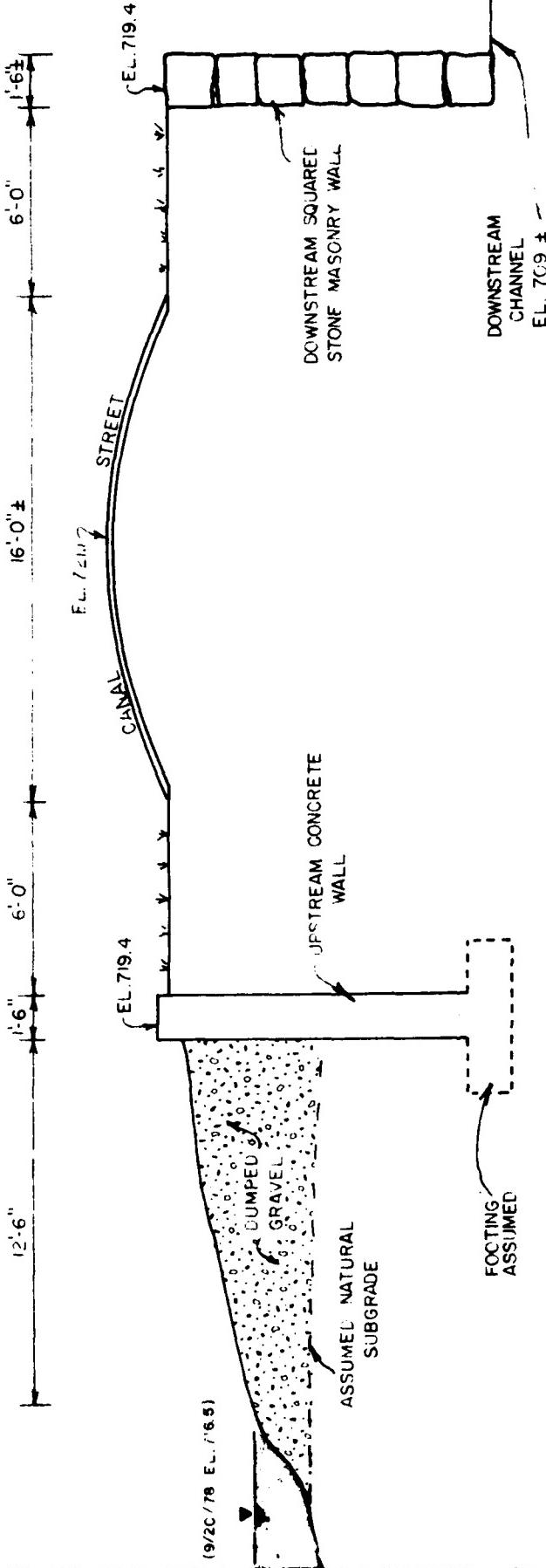
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

FIG 4

A-A

SECTION THRU GATEHOUSE

SPOFFORD LAKE	SCALE 1" = 5'
	DATE NOV. 1978



GOLDBECK, ZOMO, DUNNCLIFF & ASSOC., INC. U.S. ARMY ENGINEER DIV. NEW ENGLAND
GEOTECHNICAL CONSULTANTS CORPS OF ENGINEERS
NEWTON UPPER FALLS, MASS. WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

FIG. 6
B-B

SECTION THRU EMBANKMENT

SPOFFORD LAKE	NEW HAMPSHIRE
	SCALE 1" = 5'
	DATE NOV 1978

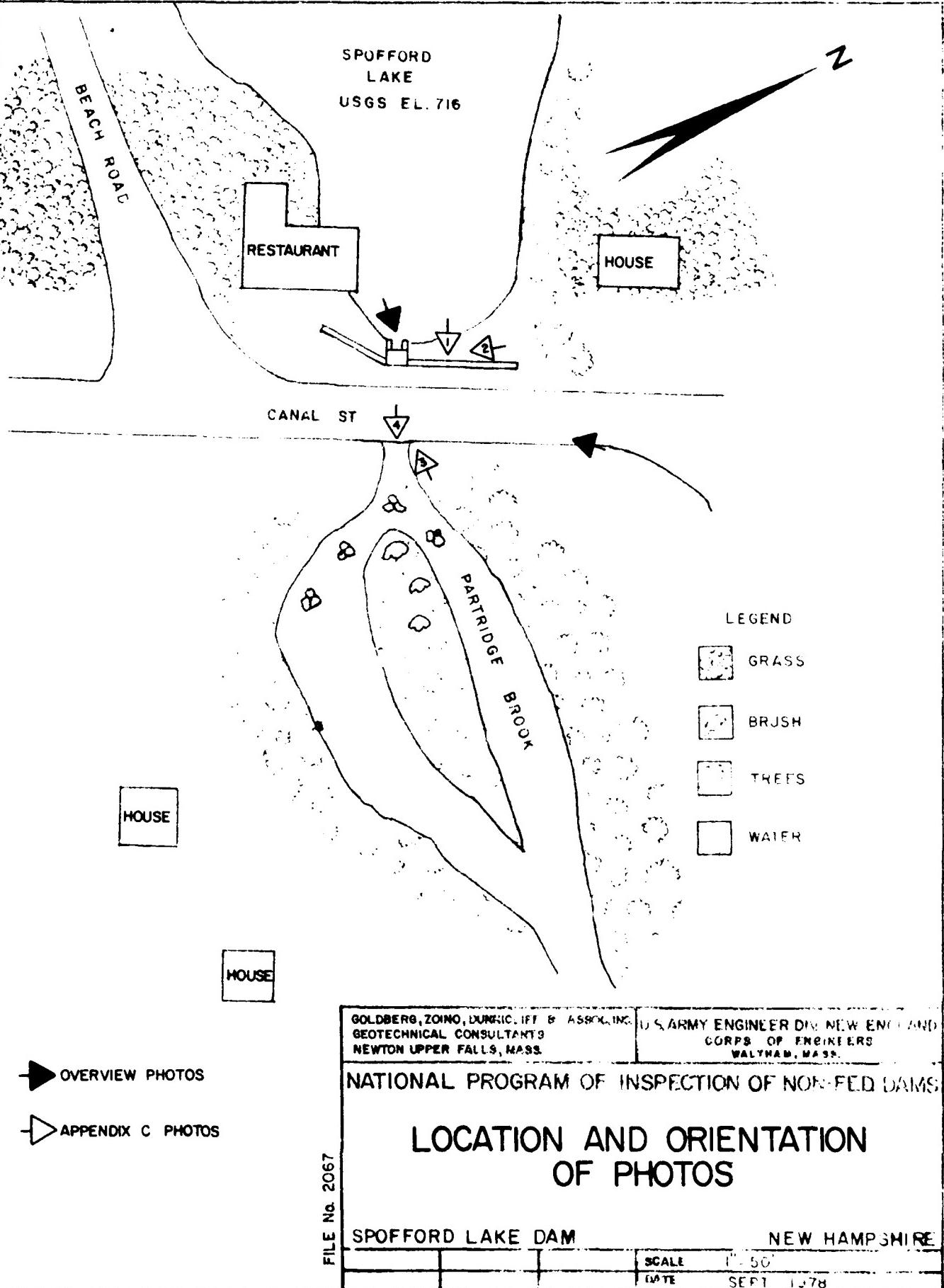
FILE No 2067

The New Hampshire Water Resources Board, 37 Pleasant Street, Concord, NH 03301 maintains a comprehensive correspondence file on the dam dating back to the 1930's. Included in this file are:

- (a) Correspondence from lake residents and other interested parties concerning both summer and winter lake levels.
- (b) A 1938 report by the New Hampshire Water Control Commission entitled "Data on Reservoirs and Ponds in New Hampshire."
- (c) A 1938 report by the same agency entitled "Data on Dams in New Hampshire."
- (d) A 1955 NHWRB drawing showing the construction of the new stoplog weir.

APPENDIX C

SELECTED PHOTOGRAPHS



C-2



1. View of upstream concrete wall showing deterioration of concrete



2. View of inlet area from left side showing erosion of concrete



3. View of outlet area in downstream squared stone masonry wall



4. View of downstream channel from road

APPENDIX D
HYDROLOGIC/HYDRAULIC COMPUTATIONS

Job 148 Dam Safety

RJH

11/2/78

1 of 14

Spofford Lake Dam COE I.D. NO. NH 00356

Size Classification = Intermediate

Hazard Classification = Significant

The Test Flood for a dam with the above classifications
is:

Test Flood = $\frac{1}{2}$ PMF to PMF

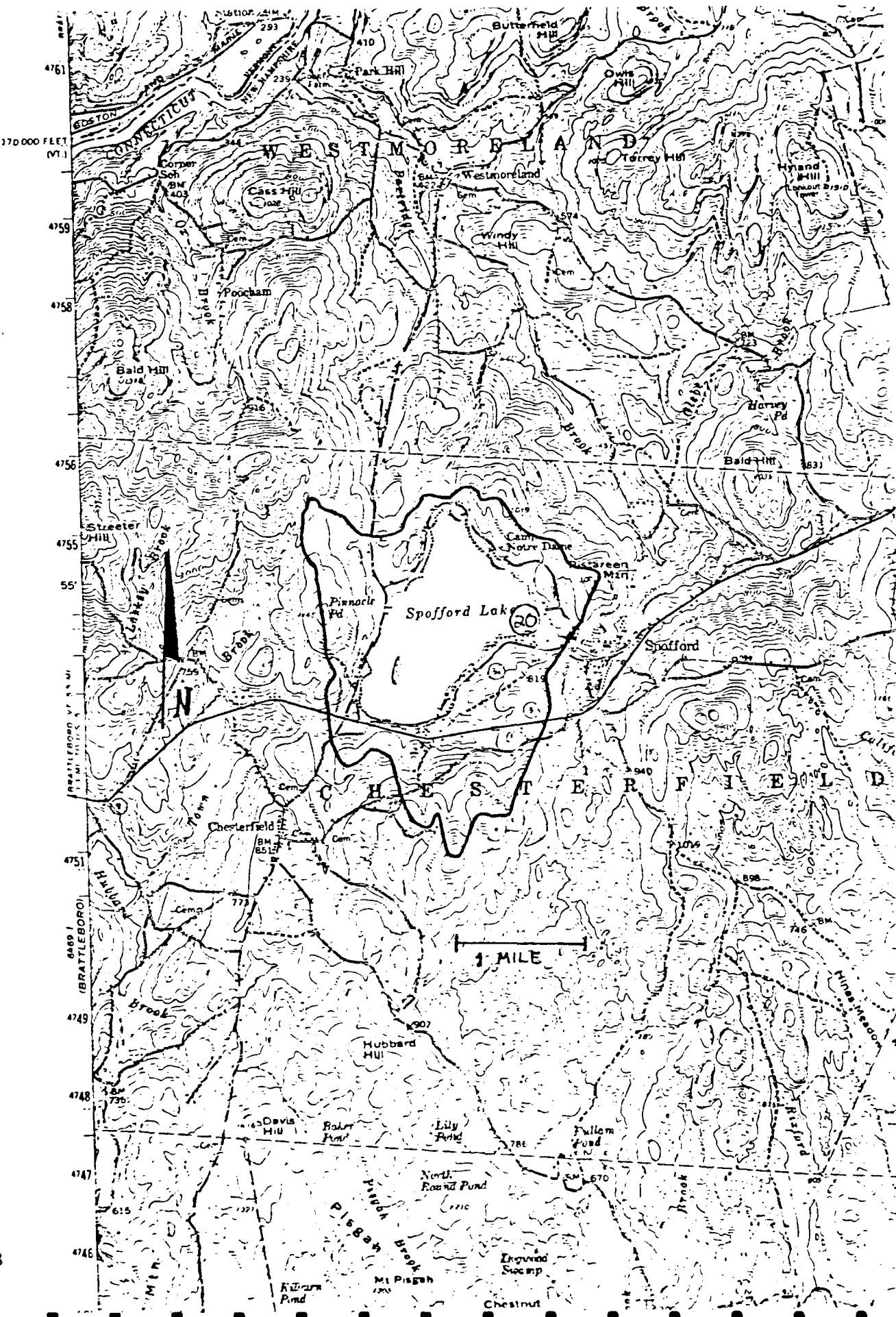
With a drainage area of 3.95 mi^2 and a topography best described as rolling, the COE curve of "Maximum Probable Flood Peak Flow Rates yields a PMF of:

$$\text{PMF} = (1925 \text{ cfs}) (3.95 \text{ mi}^2) \stackrel{\text{Size curve}}{\rightarrow} \hat{=} 7600 \text{ cfs}$$

So the range of values to choose the Test Flood from is:
 $3800 \text{ cfs} < \text{Test Flood} < 7600 \text{ cfs}$

The Test Flood should be chosen to reflect the associated risk. Therefore the Test Flood for Spofford Lake is chosen as:

Test Flood = 5500 cfs.



STORAGE-STAGE RELATIONSHIP

Surface Area of lake at normal level = 718.2 acres

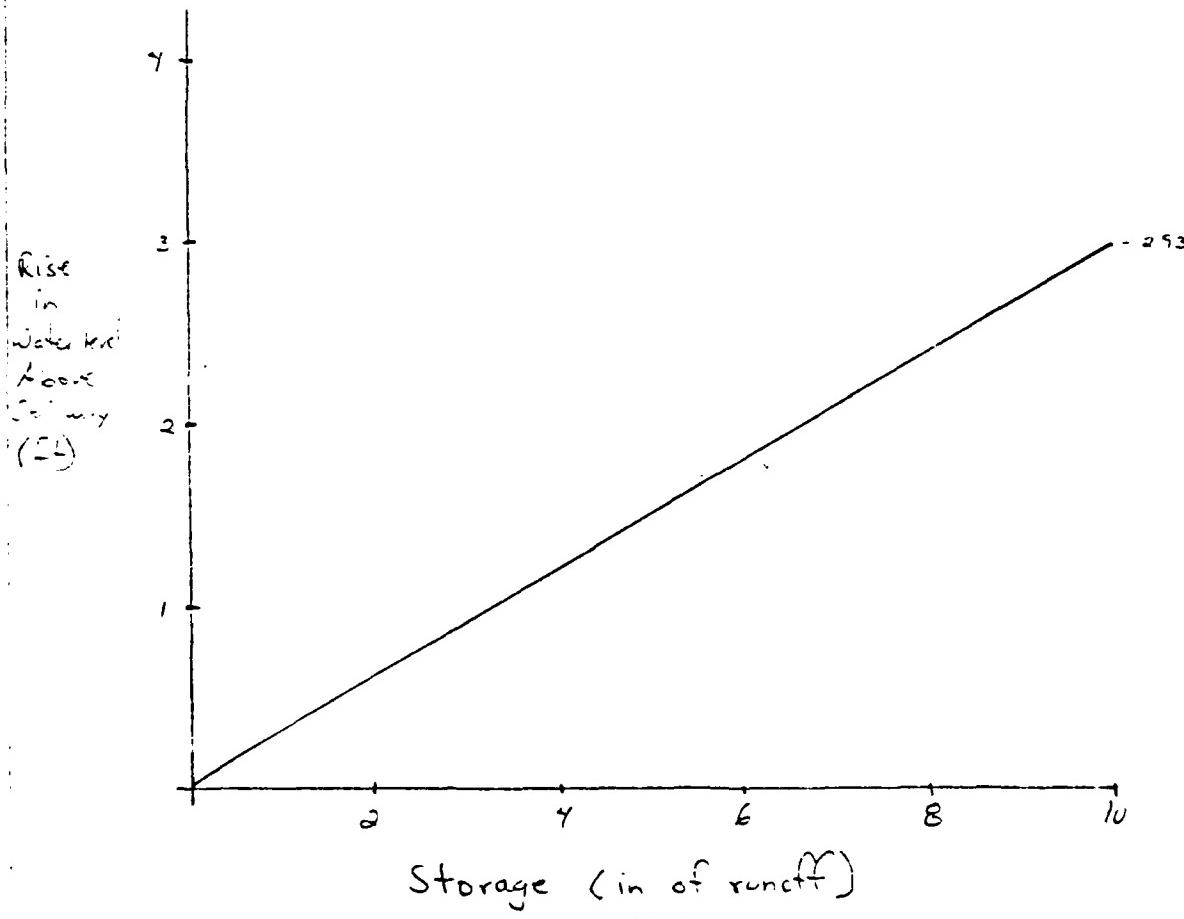
Drainage Area of lake = 3.95 sq. mi.

∴ 1" runoff yields:

$$\text{Rise in water surface} = \frac{1" (3.95 \text{ mi}^2) (640 \frac{\text{sq. ft}}{\text{mi}^2})}{718.2 \text{ acres}} = 3.52 \text{ " rise}$$

So one foot of rise results from:

$$\text{One foot} \rightarrow \frac{12"}{3.52} = 3.41" \text{ of runoff}$$



Joe & Don Eddy

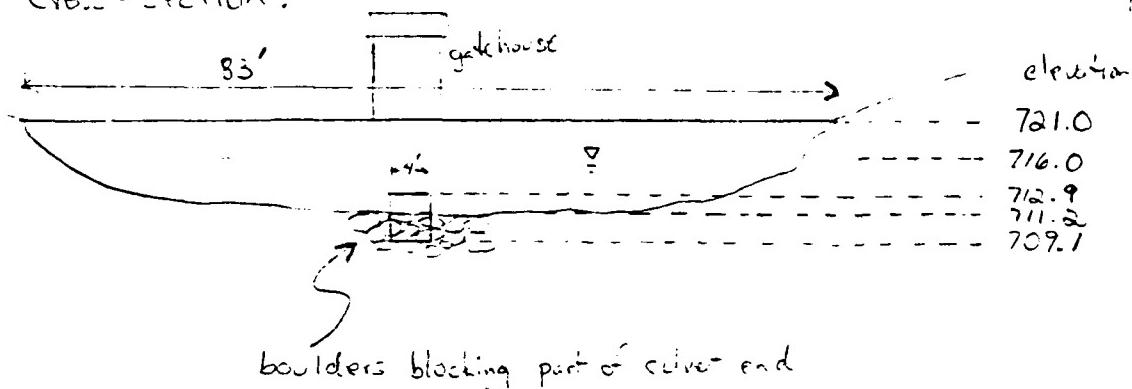
- RTH

11/3/78

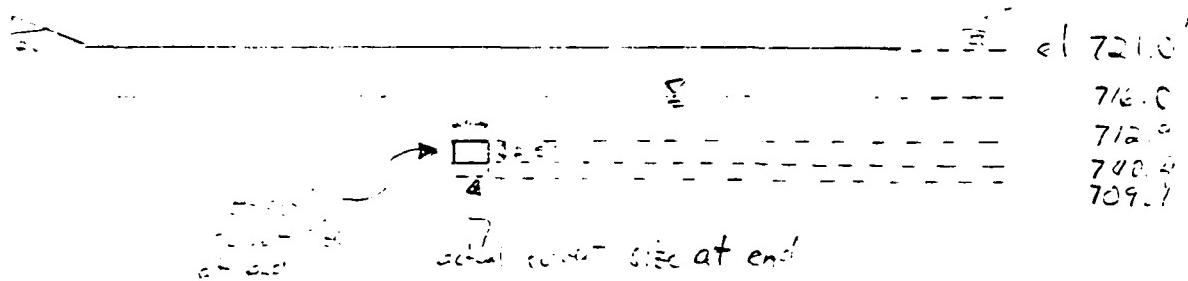
gef 14

Spartakus iste Dm

Cross-section:



مکتبہ میرزا علی



Stage-Diseases

The canal outlet is 4' by 375'. Above the inlet of the outlet is 25 m. The canal outlet is 20 m above the outlet being gradually decreasing towards the sea. The canal outlet has a width of 4' & a height of 2 m. The canal outlet has a cross-section of 4' by 0.5'. The outlet has two gates. One gate is placed to the left of the outlet & another is placed to the right of the outlet. Since the water elevation is 20 m above the outlet, the flow into the beach immediately downstream to the outlet begins because it is not to find the water rates at various elevations.

Due to the relatively small drainage area to surface area ratio ($2.3 \text{ ft}^2/\text{ft}$) and the large backwash creek overflowing (5.0 ft sec^{-1}) the entire T-1 flood can be stored. At no discharge, assuming a total tank volume of 15 inches (which is equivalent to 5 ft^3 due to 12 in. depth).

Spofford Lake Dam

To find the water surface level at the time of peak discharge the following criteria must be met:

The corrected test flood is given by:

$$(1) \quad Q = 5500 \left(1 - \frac{3.41}{H}\right)^{\frac{1}{2}} \quad H = \text{headache depth above } 716.0' \\ Q_c = \text{corrected test flood}$$

This must be consistent with the following equation (which is the flow equation for the culvert at the depths we are concerned with - constriction and tailwater controlling): * above

$$Q_c = \left[\frac{H_T^{1/2} g}{\frac{1}{A_1} + k_e + \frac{0.9 n^2 L}{R_1^{1/3} S} + \frac{1}{A_2} \left(\frac{1}{k_e} - 1 \right)} \right]^{1/2}$$

where: H_T = head differential (headwater - tailwater)

g = gravitational acceleration = 32.2 ft/sec^2

A_1 = area of culvert = 15.0 ft^2

A_2 = area at constriction = 10.0 ft^2

L = length of culvert = 32.0 ft

n = Manning's n = 0.014

R_1 = hydraulic radius = $A_1/WP = 15.0/15.5 = 0.97 \text{ ft}$ ← reflects entrance loss coef. = assumed to be 0.70 ← entrance way

k_e = entrance loss coef. = $f(A_1, A_2) = 0.741$

Inserting these values the above equation reduces to:

$$(2) \quad Q_c = 73.59 (H_T)^{1/2}$$

The flood level upstream of the dam will be given by these two equations (1) and (2) and the table of tailwaters v. flow rates.

(cont.)

Note → *— This equation is for the flow rate through the culvert with the constriction and tailwater controlling. If the headache is ~~too~~ ^{not} large the constriction factor would be the flow over the stoplogs. However the area we are concerned with, the headwater, the entrance & transition would be submerged hence the transition conditions are assumed to control.

Searched Lake Dam

where these two equations:

$$Q_1 = 5500 \left(1 - \frac{3.41}{14} H\right)$$

and

$$Q_2 = 73.59 (H_T)^{1/2}$$

 H = depth above 716.0' Q_1 = corrected flood flow H_T = head above tailwater Q_2 = culvert flow

are consistent the exact flow through the culvert (Q_2) and the headwater H will be known.

To solve this set of equations an iterative process is used:

① Assume $H = 4.0$ ft $\rightarrow Q_1 = 141.4$ cfs

enter 141.4 as Q_1 and also find the tailwater el. for 141.4 cfs (following page).

with $Q_1 = 141.4 \rightarrow H_T = 3.69'$
tailwater \rightarrow el. = d = 1.11

? $716.0' + H = 711.2 + H_T + d$
 \uparrow headwater datum \uparrow tailwater datum

$$716.0 + 4.0 \neq 711.2 + 3.69 + 1.11$$

$$720.0 \neq 716.00$$

② Assume $H = 3.90 \rightarrow Q_1 = 275.4$ cfs.

$$Q_1 = 275.4 \text{ cfs} \rightarrow H_T = 14.00 \text{ ft}$$

tailwater $\rightarrow d = 1.63$ ft

$$716.0 + 3.90 \stackrel{?}{=} 711.2 + 14.0 + 1.63$$

$$719.90 \neq 726.83$$

③ Assume $H = 3.95$ ft $\rightarrow Q_1 = 208.41$ cfs

$$Q_1 = 208.41 \text{ cfs} \rightarrow H_T = 8.02$$

tailwater $\rightarrow d = 1.40$

$$716.0 + 3.95 \stackrel{?}{=} 711.2 + 8.02 + 1.40$$

$$719.95 \neq 720.62$$

clue

Job #E Dam City

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Spofford Lake Dam

④ Assume $H = 3.96 \text{ ft} \rightarrow Q_s = 195.00 \text{ cfs}$

$$Q_s = 195.00 \xrightarrow{\text{tailwater}} H_t = 7.02 \text{ ft}$$
$$\xrightarrow{\text{tailwater}} d = 1.34 \text{ ft}$$

$$716.0 + 3.96 \text{ ft} \stackrel{?}{=} 711.2 + 7.02 + 1.34$$
$$719.96 \text{ ft} \approx 719.56 \checkmark$$

so the divert flow equals 195.0 cfs

the tailwater elevation equals 712.54 ft (USGS)

the headwater elevation equals 719.96 ft (USGS)

and the dam has 1.04 ft freeboard left before
overtopping occurs.

WPER 0.0 12.2 14.7 16.9 18.2 21.4 23.7 25.1 27.3 30.1 32.4 34.2 35.2 37.6 39.2 41.7 42.8 44.5 46.0 47.7 49.1 50.6 52.1 53.7 55.2 56.7 58.2 59.7 61.2 62.7 64.2 65.7 67.2 68.7 69.2 69.7 71.2 72.7 74.2 75.7 77.2 78.7 79.2 79.7 80.2 81.7 82.2 83.7 84.2 85.7 86.2 87.7 88.2 89.7 90.2 91.7 92.2 93.7 94.2 95.7 96.2 97.7 98.2 99.7 100.2

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የኢትዮጵያውያንድ ተስፋይ ማስተካከል ነው እና የሚከተሉት በቃላይ ተስፋይ ይችላል

Downstream of Spofford Lake 17^{km}

100

The following relationship is assumed to control between el. 711 and el. 721.0:

<u>d</u>	<u>Q</u>	<u>H_T</u>	(711.2 + d + H _T)
1	114.3	2.41	714.61
1.05	126.2	2.94	715.19
1.10	138.1	3.52	715.82
1.15	150.0	4.15	716.50
1.20	161.9	4.84	717.24
1.25	173.8	5.58	718.03
1.30	185.7	6.37	718.87
1.35	197.6	7.21	719.76
1.40	209.5	8.10	720.70
1.45	221.4	9.05	721.70

from Table (P.8)
for Reach 1 depths

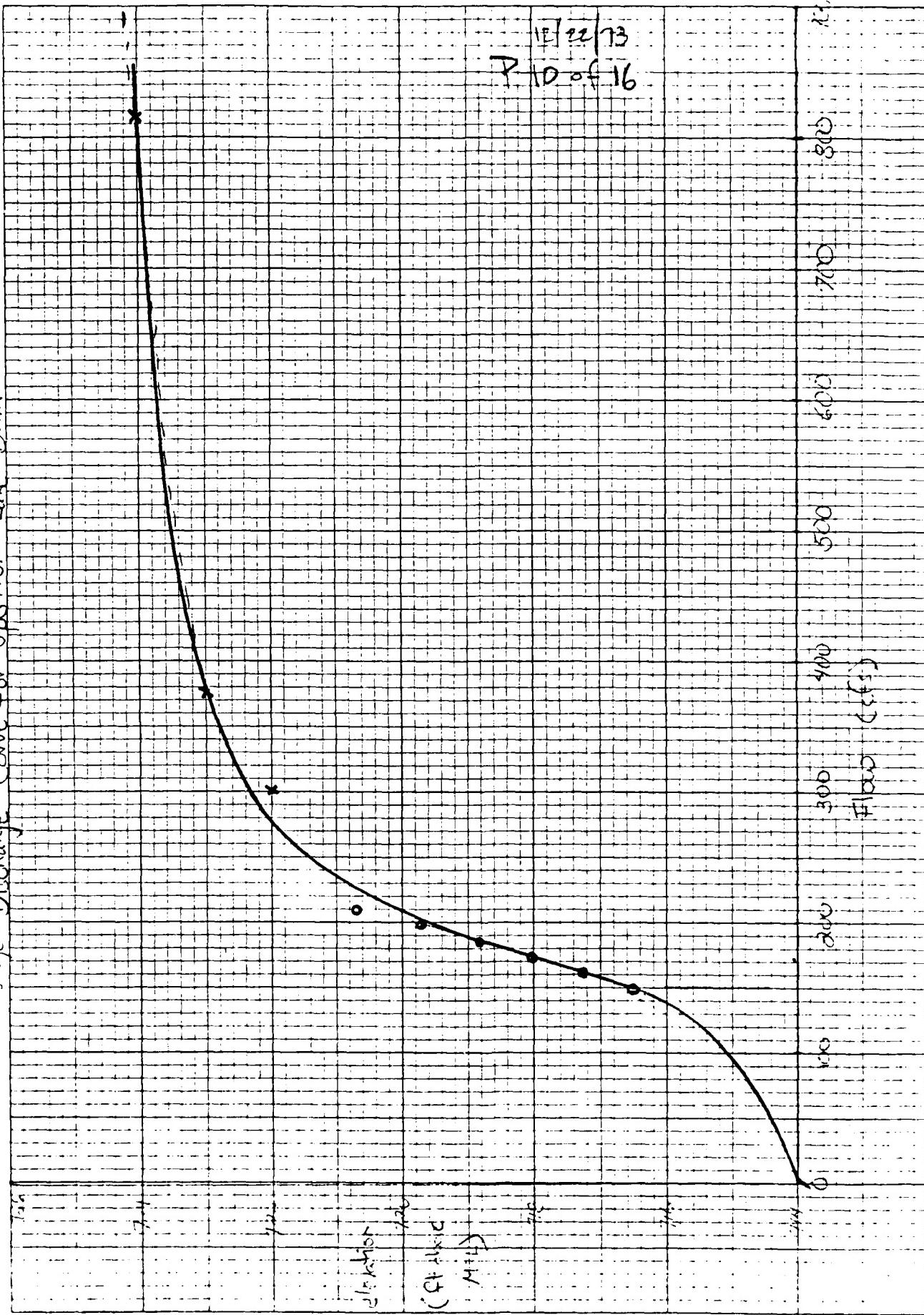
Above 722.75 the flow over the roadway is assumed to be substantial enough to cause a high enough tail water to render the sluice gate flow inconsequential.

So above 727.50 the flow equals

$$Q = 2.8 \left(83(h)^{3/2} + (20h)(.5h)^{3/2} + (50h)(.5h)^{3/2} \right)$$

where h = the height above the dam

Stage - Discharge Curve for Spofford Lake Dam



D-11

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Pelican Lake Dam

Calculations of Estimated Downstream Dam Failure Flood Stages -
Based on COE "Rule of Thumb" Guidance, April 1978.

STEP 1: Reservoir Storage at Time of Failure

Assume failure occurs when water level reaches the
top of the wall adjacent to the lake (el. = 719.4 ft)

$$\therefore \text{Storage} = \text{Normal} + \text{Surcharge} = 2800 + (719.4 - 716.0) 778 =$$

$$\text{Storage} = 5242 \text{ acre-ft}$$

STEP 2: Peak Failure Flow

$$Q_p = \frac{8}{27} w_b \sqrt{g} y_0^{3/2}$$

$$w_b = \text{breach width} < .40(83.0) = 33.2 \text{ ft}$$

$$w_b = 30 \text{ ft}$$

$$y_0 = \text{height of water above stream bed} (719.4 - 711.2) = 8.2 \text{ ft}$$

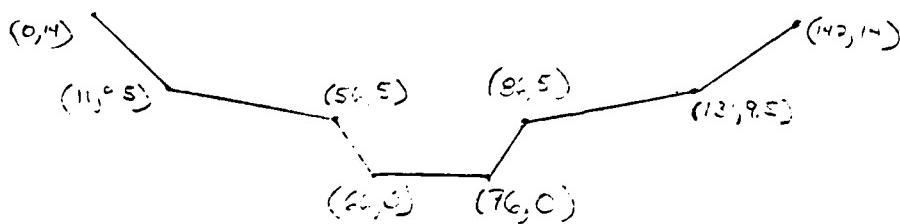
$$Q_p = \frac{8}{27} (30) \sqrt{32.2} (8.2)^{3/2}$$

$$= 1184 \text{ cfs say } \underline{1200 \text{ cfs}}$$

STEP 3: Develop Stage-Discharge Rating for Downstream Reaches

Assumed cross-sections for the downstream reaches shown
on the USGS topo are plotted below and shown on the following
page.

Reach 1: Immediately downstream of the dam
length = 800' slope = 0.017 Manning's n = 0.018

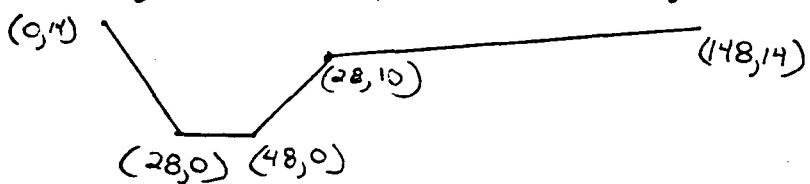


Spofford Lake Dam

Reach 2: From first reach to first bridge
 length = 800' slope = 0.008 Manning: $n = 0.025$



Reach 3: From first bridge to second bridge
 length = 700' slope = 0.008 Manning: $n = 0.018$



STEP 4: Calculate Downstream Attenuation and Flood Levels

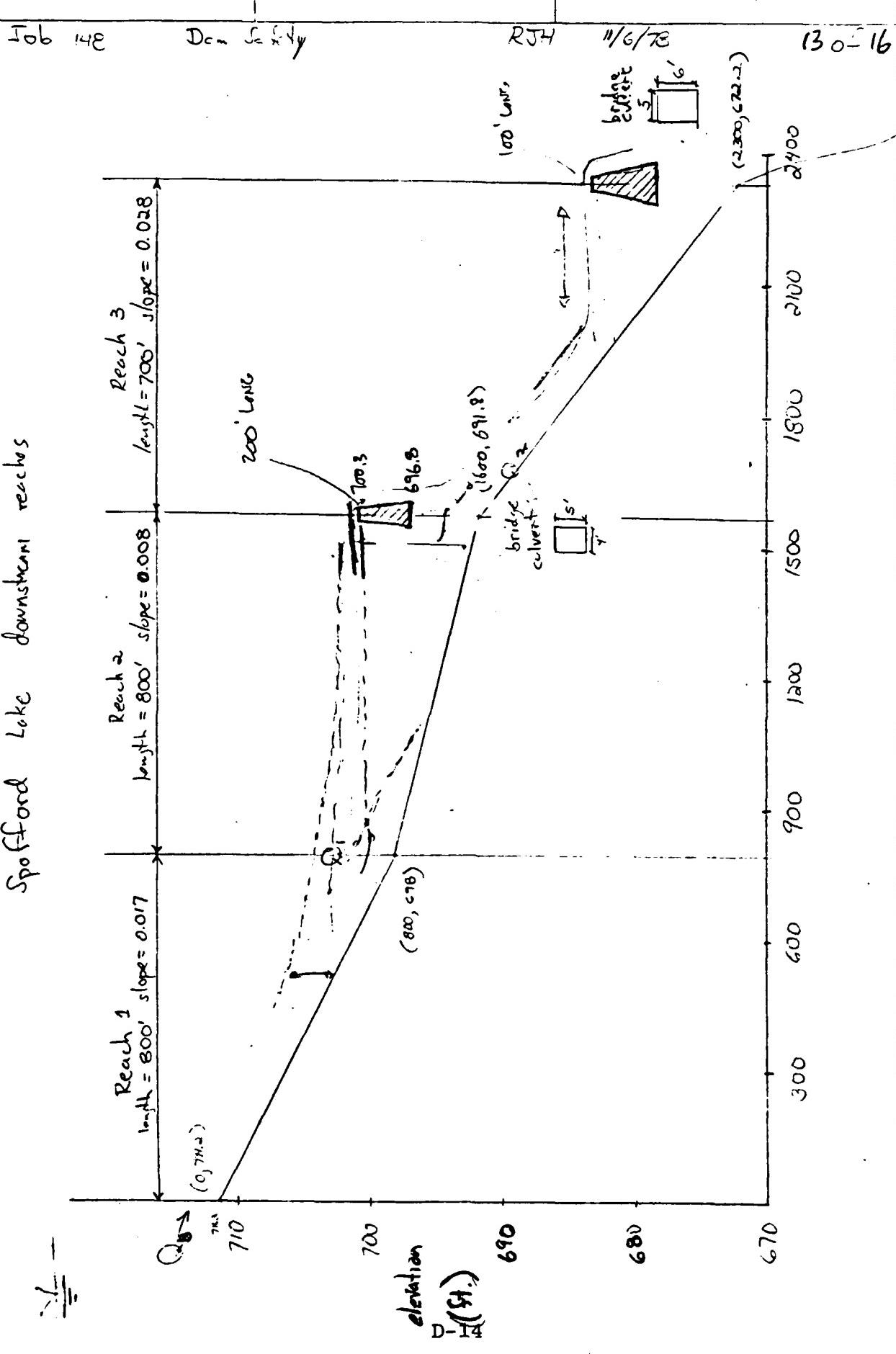
Note - Only the first reach is not controlled by downstream conditions. Both reach 2 and 3 are controlled by the bridges at their downstream terminus. Below these large reaches the stream is very steep and does not threaten any structures.

The first reach is analyzed using the equations for flow in an open channel. The computer results are shown on page eight of this series.

Reach 2 and 3, however, must include the effects of the bridge culverts. Therefore the following case is used since this presents the worst possible occurrence (short of the roadway being washed out) and therefore represents the conservative case:

It is assumed that the lake level remains relatively unchanged (as in a large lake) after the dam break and therefore the peak discharge of 1200 cfs is maintained. A steady state condition results and so each reach must also pass along 1200 cfs. The following page shows a profile of this situation.

Spoofford Lake downstream reaches



The flood levels are then found by converging on the depth that allows 1200 cfs to pass over (and through) the roadway & (culvert). The following is the analysis.

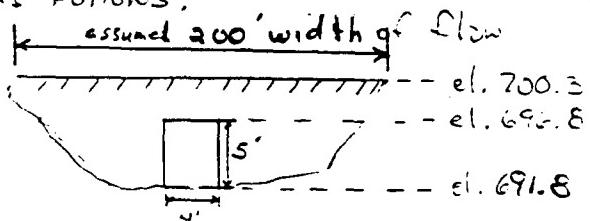
Reach 1:

$Q_p = 1200 \text{ cfs}$ which yields a depth in this reach (using the table on page seven)
of $\rightarrow 3.59 \text{ feet}$

Since we are examining the steady state case, there is no attenuation in the flow of 1200 cfs.

Reach 2: $Q_p = 1200 \text{ cfs}$

An iterative process is used to converge on the flood level in the reach. The roadway and culvert are as follows:



- ① Using the FHWA nomographs for roadway culverts and assuming the water level is at the roadway crest:

$$\text{depth to height ratio} = \frac{700.3 - 691.8}{5} = 1.70$$

$$\text{flow} = (55 \text{ cfs/ft}) 4 \text{ ft} = 220 \text{ cfs}$$

$1200 - 220 = 980 \text{ cfs}$ must pass over the roadway

Using the standard weir equation:

$$Q = 3.0 W H^{3/2} \quad W = 200' \quad Q = 980 \text{ cfs}$$

$$H = \left(\frac{980}{3(200)} \right)^{2/3}$$

$$H = 1.4 \text{ ft. (height over roadway)}$$

EE WE Dam Study

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Spartan Lake Dam

Reach 2: (cont.)

② Assume flood el. 1.4' over roadway

This gives a depth to height ratio of:

$$\frac{100.3 + 1.4 - 69.8}{5} = 1.97$$

so the culvert flow = $(61 \text{ cfs/ft})(4 \text{ ft}) = 240 \text{ cfs}$

$$1200 - 240 = 960 \text{ cfs}$$

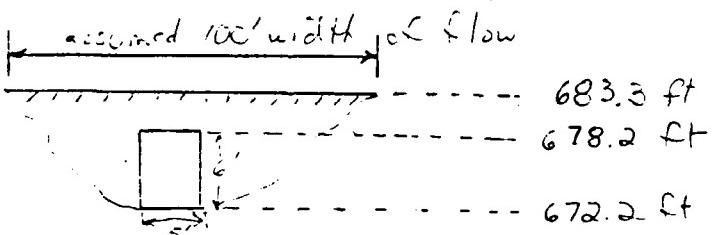
$$H = \left(\frac{960}{3(200)} \right)^{1/2} = \underline{1.40 \text{ ft}}$$

③

$$\frac{H_1 + H_2}{2} = H_{\text{avg}}$$

so the depth of flow in Reach 2
is 1.4 ft over the roadway.

Reach 3: Using the same iterative process as was used
in reach 2 with the roadway and culvert as follows:



① Assume the water level is at the roadway crest (el 683.3)

$$\text{depth to height ratio} = \frac{683.3 - 672.2}{6} = 1.85$$

$$\text{culvert flow} = 78(\text{cfs/ft})(5 \text{ ft}) = 390 \text{ cfs}$$

$1200 - 390 = 810 \text{ cfs}$ must pass over the top of the
roadway

$$H = \left(\frac{810}{3(100)} \right)^{1/2} = \underline{1.9 \text{ ft}}$$

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② Assume the water level is 1.9 ft. above the roadway.

$$\text{depth to height ratio} = \frac{683.3 + 1.9 - 672.2}{6} = 2.17$$

$$\text{current flow} = (0.1 : f_{sf}) 5 \text{ ft} = 405 \text{ cfs}$$

$$1200 \text{ cfs} - 405 = 795 \text{ cfs}$$

$$H = \left(\frac{795}{3(100)} \right)^{1/2} = 1.9 \text{ ft}$$

③

$$\text{so } H_{Av3} = \frac{H_1 + H_2}{2}$$

$$H_{Av3} = 1.90 \text{ ft}$$

The depth behind the roadway in Reach 3 is
1.90 ft over the roadway

Summary

<u>Reach</u>	<u>Depth (ft)</u>	<u>(USGS) flood el.</u>
1	3.59	
2	9.86	701.86
3	13.00	685.20

APPENDIX E
INFORMATION AS CONTAINED IN
THE NATIONAL INVENTORY OF DAMS

INVENTORY OF DAMS IN THE UNITED STATES

STATE NUMBER	IDENTITY DIVISION	STATE	COUNTY	CONG. DIST.	NAME	LATITUDE (NORTH)	LONGITUDE (WEST)	REPORT DATE
NH 354	RED	14	004	02	SPOFFORD LAKE DAM	42°54' N	72°25' W	27 NOV 78

POPULAR NAME		NAME OF IMPOUNDMENT	
		SPOFFORD LAKE	

REGION BASIN	RIVER OR STREAM	NEAREST DOWNSTREAM CITY-TOWN-VILLAGE		DIST. FROM DAM (MIL.)	POPULATION
01 NB	PARTIDGE BROOK	SPOFFORD		0	1800

TYPE OF DAM	YEAR COMPLETED	PURPOSES	STATIC HEAD (FT.)	HYPDRAULIC HEAD (FT.)	IMPOUNDING CAPACITIES (ACRE-FT.)	OWNER	POWER CAPACITY (KWH/HR.)	INSTALLED (KWH/HR.)	PROPOSED (KWH/HR.)	NAVIGATION LOCKS	MAINTENANCE
RECTRAG	1919	RC	11	11	6460	NED	2400	2400	2400	N	N

REMARKS

DIS HAS LENGTH	SPILLWAY TYPE	MAXIMUM DISCHARGE (FT.)	VOLUME OF DAM (CY)	POWER CAPACITY (KWH/HR.)	INSTALLED (KWH/HR.)	PROPOSED (KWH/HR.)	NAVIGATION LOCKS
2	AS	N					

OWNER	ENGINEERING BY	CONSTRUCTION BY	REGULATORY AGENCY	OPERATION	Maintenance
TOWN OF CHESTERFIELD					

DESIGN	CONSTRUCTION	INSPECTION BY	INSPECTION DATE	AUTHORITY FOR INSPECTION
NH WAT RES BD	NH WAT RES BD	GOLDBERG ZDINO DUNNICLIFF + ASSOC	215 FPT 78	PUBLIC LAW 92-367

REMARKS	

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